Mountain Hotel, Urban Virginia

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Structural Option

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Table of Contents

Executive Summary

After a thorough review of the building documentation it was demonstrated that loads and values used for design of the Mountain Hotel were reproducible within close percentages. Herein also is a description of the building purpose and site, including a more detailed analysis of structural systems and applicable codification thereof. There is also a demonstration of load paths which this buildings structure uses to take loads back to the earth.

Building Introduction

Figure 3-1
An aerial view from bing.com maps with the building superimposed on. Hotel is in Red, Garage in Yellow.

The new hotel is to be located in a wealthy urban area of Virginia (Location shown in Figure 3-1). The site chosen for construction of the new hotel is a prominent location previously occupied by a chain of parking lots, which border the main street of the town.

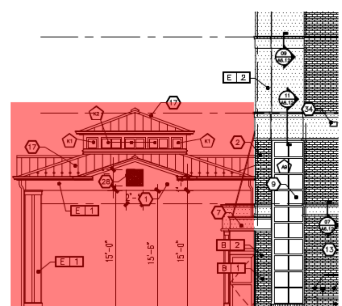


Figure 3-2
Porte Cochere attached to Hotel over main Entrance.

In order to match the new building into its surrounding architecture the first two floor facades are brick with large glazing panels, while the upper facade uses a palette of varying shades from brick red to white which enables it to match the brick and concrete of the surrounding buildings, including the adjacent concrete parking structure. However, in place of the brick or concrete, the upper stories of the hotel use a lighter more cost effective cladding, exterior insulation finishing system (EIFS) panels. The Porte Cochere on the west side will help funnel visitors into the main lobby where they can check-in and be directed to their rooms, other amenities, or sites of the town.

Guest rooms are located on the second through sixth floors totaling just over 40,000 square feet. Though the main function is to appease guests with a home away from home, it also contains meeting rooms for conferences, offices for hotel management, and a 40,000 square foot parking garage. Total building area is approximately 120,000 square feet.

Structural Overview

The hotel rests on reinforced concrete spread footings ranging from 12 to 42 inches in depth. Concrete piers transfer the load into the interior footings from the steel columns. The exterior concrete basement walls rest on strip footings, ranging from 12 to 24 inches, are load bearing and double as sheer walls for the lateral system. A500 Grade B hollow structural steel ranging from four to 16 inches, longer dimension, is used for the superstructure columns. Some of the floors are supported by wide flange beams, ranging from W8 to W21, while others are resting on steel stud bearing walls as shown in Figure 4-1. The lateral system employs a combination of reinforced concrete shear walls, specially reinforced masonry shear walls and light framed wall system with flat strap bracing extending from the ground floor to roof level in both the long and short directions. Floors ground through six are installed as a series of eight inch precast hollow-core planks ranging in length from 9' 2" to 25' 8". The roof is also built of four or eight inch hollow-core planks. Both the brick walls and EIFS system are attached to cold formed steel stud walls. The loading on the exterior facade is transferred through the wall framing to the floors and into the lateral system.

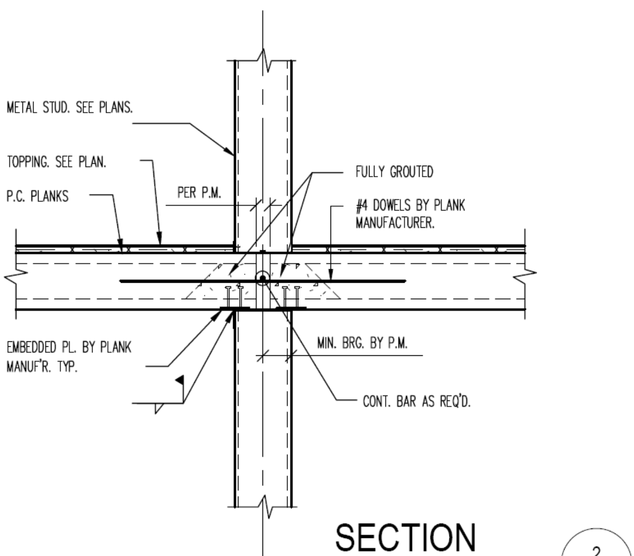


Figure 4-1
Hollow core plank ends spliced and resting on light gauge steel stud framed walls.

The garage is also supported on reinforced concrete spread footings 12 to 30 inches in depth, and strip footers 12 to 24 inches in depth. Piers transfer the load into the footings from the columns and the walls rest directly on the strip footings. Piers and beams are poured monolithic with the walls. Columns support two-way slabs and utilize drop panels, and edge beams.

Code Requirements

Standards and codes governing construction are as follows:

2009 ICC/ANSI A117.1

2009 International Building Code

2009 Virginia Uniform Statewide Building Code

2008 NEC – National Electric Code

2009 ICC – International Mechanical Code

2009 ICC – International Plumbing Code

2009 ICC – International Energy Conservation Code

- All concrete work shall be in accordance with ACI 301, ACI 318 and ACI 302 latest editions.
- All Masonry work shall be in accordance to: ACI 530/ASCE 5, "Building code requirements for Masonry structures"; ACI 530/ASCE 6, "specifications for masonry structures"
- Structural Steel Shall conform to the AISC "Specification for the design fabrication and erection of structural Steel for buildings", Latest edition, except chapter 4.2.1, code of standard practice
- All light gauge framing shall conform to "the specification for the design of cold-formed steel structural members", latest edition, by AISI
- All Wood framing shall conform to the "national design specification for wood construction" latest edition, published by the national forest products association,
- In addition to the requirements included in these structural notes, all construction and materials shall further conform to the applicable provisions of the following standards:
    1. American Society for Testing and Materials (ASTM)
    2. American Concrete Institute (ACI)
    3. National Concrete Masonry Association (NCMA)
    4. American Institute of Steel Construction (AISC)
    5. American Welding Society (AWS)
    6. American Iron and Steel Institute (AISI)
    7. Steel Structures Painting Council (SSPC)
    8. American Forest and Paper Association
    9. National Forest Products Association (NfoPA)

Governing the Parking Garage is all of the above with the exception of:

2006 International Building Code

2006 Virginia Uniform Statewide Building Code

Gravity System

Superstructure

This building uses several types of structural members to carry the various gravity induced loads to the earth. The hotel roof and all above grade floors utilize hollow core planks to support the dead loads of the structure as well as all the amenities people and other items. The planks typically rest on cold-formed steel stud shear walls which pass the load onto the floor below, and so on until it either reaches either a reinforced concrete shear wall or a wide flange beam. W-shapes made to the ASTM standard A992 range in size from W6x15 to W33x130. ASTM A500 Hollow Structural Section (HSS) columns hold the beams in place. Most of the HHS columns terminate in the lower floors; however there are several members that transfer load directly from the roof into the foundations. The Elevator and stair towers are an exception the typical framing types. They use specially reinforced masonry sheer walls to resist both gravity and lateral loads stretching from above the normal roof height and down into the foundation.

Substructure

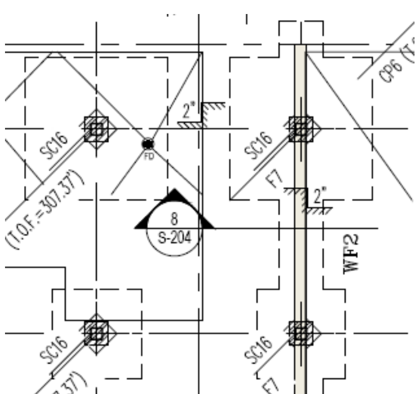


Figure 6-1
Shows a portion of the foundation plan using various structural elements.

The substructure uses a series of reinforced concrete shear walls to transfer the loads from the superstructure into the wall footings of the foundation (Figure 6-1). Under columns and column piers, there is a series of spread footings the largest of which is 16”x16”x42”deep. Footings maintain a minimum compressive strength of 3000psi, which is the same as the maximum bearing pressure for the soil atop which they rest. Other concrete members have an Fc of 5000psi.

Snow Loads

Snow Load Requirements		
	As Designed	Per ASCE7-10
N-S Direction		
Flat Roof	21psf	25psf
Drift Surcharge	Not listed	58.5psf
E-W Direction		
Flat Roof	21psf	25psf
Drift Surcharge	Not listed	34.3psf

Figure 7-1
Comparing Snow Load Requirements for as designed verses the ASCE7-10 Provisions

Because of its northern location snow loads can potentially produce a controlling load case. The use of tall parapet walls to conceal the rooftop equipment creates a compounded risk for large loads resulting from drifts. This case was examined in some detail using provisions outlined in chapter 7 of ASCE7-10 (Appendix B). The designers of this hotel used provisions from ASCE7-05 and therefore the results shown in Figure 7-1 suggest higher loading requirements.

Typical Gravity Loads

Loads were checked over a single load path using LRFD load combinations (Appendix B) from the roof to the foundation. Members were within a 5% suggesting the designers may have employed Strength Design as well.

Lateral System

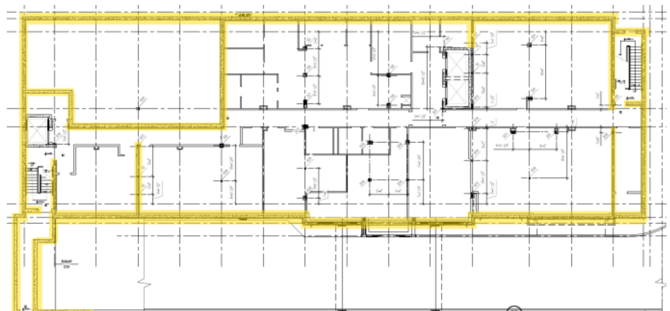


Figure 8-1
Locations of various Shear Walls

Below grade, lateral forces are resisted through a system of reinforced concrete shear walls some of which are highlighted in Figure 8-1. They range in thickness from eight to 14 inches. A few of these walls extend up to the second or third stories, but most of the superstructure employs cold-formed steel stud walls with flat strap bracing to resist wind and earthquake loadings. Locations are shown in Figure 8-2. The elevator and stair towers also contain specially reinforced masonry shear walls to resist forces in the building's shorter dimension.

Wind Loads

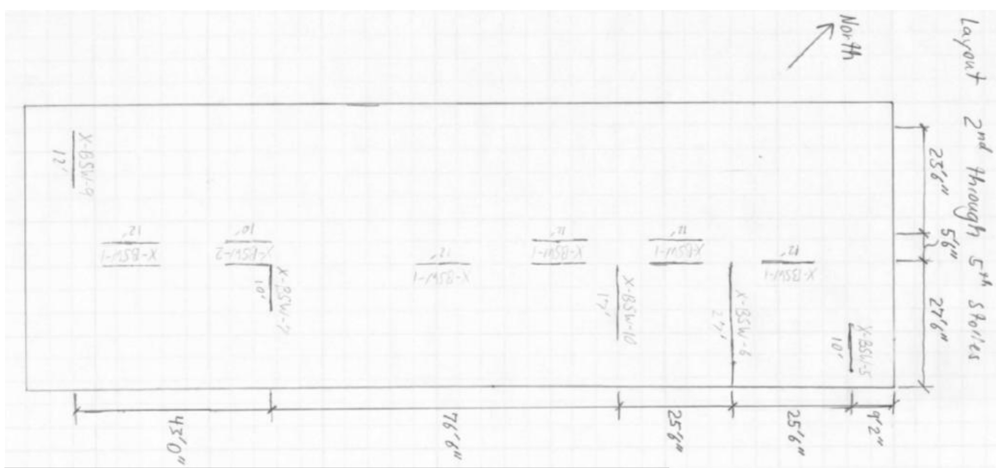


Figure 8-2
Typical locations for light gauge shear stud walls with flat strap bracing.

The wind loads on the building are collected by the exterior facade. The cladding, EIFS or brick, is mounted to the cold-formed stud walls. These transfer the wind loads into the floors which are resisted by the main lateral force resisting system. The upper floors utilize light gauge shear walls with flat strap bracing and transfer the load down through the lower walls into reinforced concrete shear walls. The concrete walls are joined to their footings and the load is manifested as a moment in the foundation. This moment is finally resisted by a couple induced by the ground bearing pressures and the weight of the structure.

Basic wind loads for the hotel were calculated using the main wind force resisting system directional procedure outlined per chapter 27 of ASCE7-10. Several assumptions were made in order to use this procedure. First building was presumed to be rectangular with no depressions or extrusions in the facade. The parapet height was considered to be a constant nine foot height for the entire perimeter, and the shallow slope of the grade was considered to be negligible.

Wind pressures including windward, leeward, sidewall, and internal pressure were determined as shown in Figure 9-1 (N-S) and Figure 9-3 (E-W). Calculations and diagrams can be found in Appendix B. Excel was used to tabulate the story forces in each direction (Figure 9-2 (N-S) and Figure 9-4 (E-W)) and a total overturning moment was found for each direction (also in Figures 9-2 and 9-4). A more thorough analysis of wind pressures including that on the components and cladding will be included in Technical Report Three.

Normal to 62ft Wall — Width = 62

Elevation	Windward Pressure	Leeward Pressure	Internal Pressure	Total Pressure
71	18.2	5.69	4.82	28.71
70	17.4	5.69	4.82	27.91
60	16.6	5.69	4.82	27.11
50	15.8	5.69	4.82	26.31
40	14.9	5.69	4.82	25.41
30	13.7	5.69	4.82	24.21
25	12.9	5.69	4.82	23.41
20	12.1	5.69	4.82	22.61
15	11.2	5.69	4.82	21.71

Figure 9-1
Wind pressures in the North – South direction

Figure 9-2
Story Forces in North – South direction
With base ear and overturning moment

Level	Story Height	Story Force
Roof	62	23706.475
6th	52.5	15496.59
5th	43	14572.635
4th	34	13884.435
3rd	24.5	14967.637
2nd	13.3	16488.745
Base Shear		99116.517

Total Moment 22587684

Normal to 190ft Wall — Width = 190

Elevation	Windward Pressure	Leeward Pressure	Internal Pressure	Total Pressure
71	18.2	5.69	4.82	28.71
70	17.4	5.69	4.82	27.91
60	16.6	5.69	4.82	27.11
50	15.8	5.69	4.82	26.31
40	14.9	5.69	4.82	25.41
30	13.7	5.69	4.82	24.21
25	12.9	5.69	4.82	23.41
20	12.1	5.69	4.82	22.61
15	11.2	5.69	4.82	21.71

Figure 9-3
Wind pressures in the East – West direction

Figure 9-4
Story Forces in East – West direction
With base ear and overturning moment

Level	Story Height	Story Force
Roof	62	72648.875
6th	52.5	47489.55
5th	43	44658.075
4th	34	42549.075
3rd	24.5	45868.565
2nd	13.3	50530.025
Base Shear		303744.17

Total Moment 54886429

Earthquake Loads

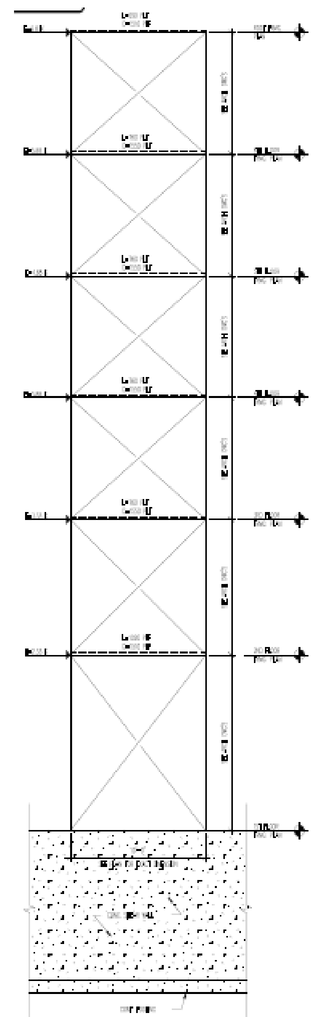


Figure 10-1
Bracing showing design to resist seismic forces

Earthquake loads, are actually displacements induced as the mass of a structure attempts to regain equilibrium as the earth moves underneath it. These displacements are resisted by the main lateral force resisting system which sends the counter displacements, due to the momentum of the structure, back to the ground. In order to quantify the strength needed to resist these displacements the response force induced in the structure is determined using historical data (an example of a designed frame shown in Figure 10-1).

Earthquake loads for the hotel were calculated using the equivalent lateral force procedure outlined per chapters 11 and 12 of ASCE7-10. The assumption was made that since the weight of the walls was significantly less than that of the floors, the wall weights are negligible and can be ignored in the seismic calculations.

An overall building base shear was determined and used to find story forces at each level. Calculations can be found in Appendix B. A more thorough analysis of earthquake loading will be included in Technical Report Three.

Conclusion

The Mountain Hotel combines typical braced frame construction with cold-formed steel stud shear walls to create an efficient solution to this buildings structure. It adequately reflects both typical hotel design through is use of stud framing, and situates itself comfortably into an urban environment though its height and architecture. It also demonstrates that flat back braced cold-formed steel stud sheer walls can be an astounding alternative to typical braced frames and moment frames in a low earthquake risk environment. Compounded with the use of hollow core planks significantly lightens the overall weight of the building.

Appendix A – Design Loads

Roof Loads					
Snow Load					
	Ground Snow Load, Pg			30	PSF
	Flat Roof Snow Load, Pf = Ce*Pg			21	PSF
	Snow Exposure Factor, Ce			0.7	
	Snow Load Importance Factor, I			1.0	
	Snow Drift & Sliding Surcharge:				Per Code Requirements
Roof Load					
	Roof Live Load (Horizontal Projection)			37	PSF minimum
	Dead Load			81.5	PSF
Floor Loads (PSF)					+ Mechanical Unit Weight per MEP Drawings
		Dead Load	Live Load	Total Load	
Living Areas		81.5	40	121.5	
Common Areas		75.5/81.5	100	175.5/181.5	
Stairs		50	100	150	
Storage (Light)		81.5	125	206.5	
First Floor Only		106.5	100	206.5	
Wind Loads					
Basic Wind Speed (3- Second Gust)				90	MP
Occupancy Category				II	
Wind Importance Factor, 1				1.0	
Wind Exposure				B	
Internal Pressure Coefficient, Gcpi				±0.18	
Components and Cladding				21	PSF
Earthquake Loads					
Seismic Importance Factor, 1				1.0	
Mapped Spectral Response Accelerations				Ss = 15.5	% g
				S1 = 5.1	% g
Site Class				D	
Spectral Response Accelerations				Sds = 16.5	% g
				Sd1 = 8.2	% g
Seismic Design Category				B	
Design Base Shear				208	
Seismic Response Coefficients				0.028	
Response Modification Factors				4	
Foundation					
Footing Design Soil Bearing Pressure				3000	PSF
Back Fill Material Equivalent Fluid Pressure				55	PSF/FT
Deflection Limits					
			Live Load	Total Load	
Floor			SPAN/480	SPAN/360	
Floor Under Ceramic Tile			SPAN/720	SPAN/360	
Roof Trusses			SPAN/360	SPAN/240	
Roof Rafters			SPAN/240	SPAN/180	
Ceiling Joist			SPAN/360	SPAN/240	
Roof Ridge/Beam			SPAN/360	SPAN/120	

Appendix B – Gravity Loads

Snow Loads | Tech 1

Using ASCE 7-10

Figure 7-1 → $p_g = 25^{psf}$

Table 7-2 → $C_e = 1.0$

Table 7-3 → $C_t = 1.0$

Importance Factor → 1.0

eq 7.3-1 → $p_f = .7 C_e C_t I_s p_g = (.7)(1.0)(1)(1)(25^{psf}) = 17.5^{psf}$

l_u in long direction $= 190^{ft}$
in short direction $= 62^{ft}$

Figure 7-9 → $h_{d(190)} = .43\sqrt[3]{190}\sqrt[4]{25+10} - 1.5 = 4.51^{ft} \times .75 = 3.38^{ft}$

$h_{d(62)} = .43\sqrt[3]{62}\sqrt[4]{25+10} - 1.5 = 2.64^{ft} \times .75 = 1.98^{ft}$

eq 7.7-1 → $\gamma = .13 p_g + 14 = .13(25) + 14 = 17.3^{lb/ft^3}$

$h_b = p_f/\gamma = 17.5/17.3 = 1.01^{ft}$

$h_c = 9^{ft} - 1.01^{ft} = 7.99^{ft} \rightarrow 8^{ft}$

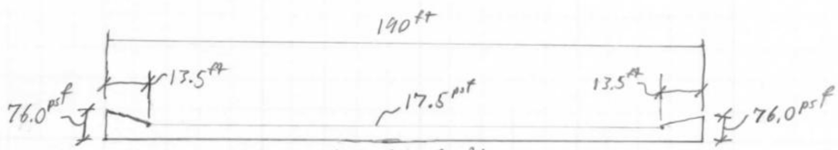
→ (drift width) $w_{(190)} = 4(3.38) = 13.5^{ft}$

$w_{(62)} = 4(1.98) = 7.92^{ft}$

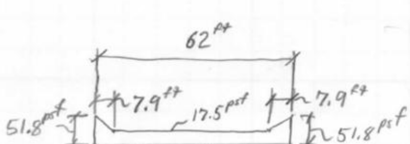
drift surcharge

$p_{d(190)} = 3.38^{ft}(17.3^{lb/ft^3}) = 58.5^{psf}$

$p_{d(62)} = 1.98^{ft}(17.3^{lb/ft^3}) = 34.3^{psf}$



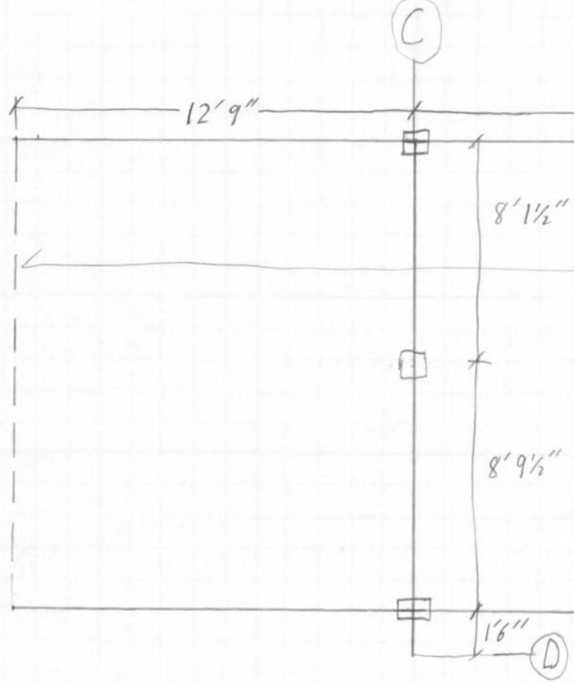
North-South Section



East-West Section

Gravity Check	Tech 1	

Column Line 8 between C & D



Roof Loads: $1.2D + 1.6L_r = 1.2(81.5^{psf}) + 1.6(37^{psf}) = 157^{psf} \times 25'6'' = 4^{k/ft}$

Floor Loads: $1.2D + 1.6L = 1.2(81.5) + 1.6(40) = 161.8^{psf} \times 25'6'' = 4.13^{k/ft}$

Walls negligible weight, strength → roof → W8 = 33ksi @ 16″ O.C. 800S200-43
6th → W10 = 50ksi @ 16″ O.C. 800S200-68
5th → W11 = 50ksi @ 16″ O.C 800S200-97
4th → W12 = 50ksi @ 16″ O.C. 800S200-97x2
3rd → W12 = 50ksi @ 16″ O.C.
2nd → SC5 & W12
1st → SC5 & W16x50 → 10′ → 276k-ft
B → SC5 → HHS8x8x3/8 → @12′ → 372k, 14′ → 353k
Base Plate 3/4″x14″x14″ → 1′4″ below
Footing 31′x9′x28″ Longitudinal Bars
Top: #6 @ 12″ O.C.
Bottom #7 @ 12″ O.C.
Transverse Bars
Top #4 @ 12″ O.C.
Bottom #6 @ 12″ O.C.

Gravity Check | Tech 1

Roof $= 4^{k/ft}$

$6^{th} = 4.13 + 4 = 8.13^{k/ft}$

$5^{th} = 2(4.13) + 4 = 12.26^{k/ft}$

$4^{th} = 3(4.13) + 4 = 16.39^{k/ft}$

$3^{rd} = 4(4.13) + 4 = 20.52^{k/ft}$

$2^{nd} = 5(4.13) + 4 = 24.65^{k/ft}$

$1^{st} = 6(4.13) + 4 = 28.78^{k/ft} \rightarrow \frac{w\ell^2}{8} = \frac{(28.78)(8'9\frac{1}{2}")^2}{8} = 278^{k\text{-}ft} > 276^{k\text{-}ft}$ NG $< 5\%$

$B = (28.78^{k/ft} + .05) \times \left(\frac{8'9\frac{1}{2}" + 8'1\frac{1}{2}"}{2}\right) = 243.9^k < 353^k$ ✓

footing $\rightarrow \frac{243.9^k}{(14" \times 14")} = 1{,}244^{psi} < 3000^{psi}$ ✓

Page 2 of 2

Appendix C – Lateral Loads

Wind Calcs Tech 1

Basic Wind Speed $V = 115^{mph}$ (ASCE7-10 Figure 26.5-1A)

Building Classification → Occpancy Category II → Importance Factor 1.0
(ASCE7-10 Table 1.5-2)

Exposure → B →

Velocity Pressure

$q_z = .00256 K_z K_{zt} K_d V^2$

MWFRS → K_z: $0\text{-}15^{ft} \rightarrow .57$
$15\text{-}20^{ft} \rightarrow .62$
$20\text{-}25^{ft} \rightarrow .66$
$25\text{-}30^{ft} \rightarrow .70$
$30\text{-}40^{ft} \rightarrow .76$
$40\text{-}50^{ft} \rightarrow .81$
$50\text{-}60^{ft} \rightarrow .85$
$60\text{-}70^{ft} \rightarrow .89$
$70\text{-}80^{ft} \rightarrow .93 = K_h$

$K_{zt} \rightarrow 1.0$

$K_d = .85$

$q_z = .00256 K_z (1.0)(.85)(115^{mph})^2 = 28.7776 K_z$

Design Wind Pressures ±.18 → Enclosed

$p = q\,\overset{.85}{G} C_p - q_i (GC_{pi})$

Windward

C_p → Figure 27.4-1

$p = 28.7776 K_z (.85)(.8) - 28.7776(.93)(\pm.18)$
$= 19.57 K_z \pm 4.82$

(All ±4.82 for internal Pressure)

$p_{15} = 19.57(.57) = 11.15^{psf}$
$p_{20} = 19.57(.62) = 12.13^{psf}$
$p_{25} = 19.57(.66) = 12.92^{psf}$
$p_{30} = 19.57(.70) = 13.70^{psf}$
$p_{40} = 19.57(.76) = 14.87^{psf}$
$p_{50} = 19.57(.81) = 15.85^{psf}$
$p_{60} = 19.57(.85) = 16.63^{psf}$
$p_{70} = 19.57(.89) = 17.42^{psf}$
$p_{80} = 19.57(.93) = 18.20^{psf}$

Wind Calcs | Tech 1

Sidewall: $C_p = -.7 \rightarrow p = 28.7776(.93)(.85)(.7) = 15.9^{psf}$

Leeward

Normal to 190′ wall $\rightarrow L/B = .326 \rightarrow C_p = -.5$

Normal to 62′ wall $\rightarrow L/B = 3.06 \rightarrow C_p = -.25$

$p = 28.7776(.93)(.85)\,C_p = 22.7\,C_p$

Normal to 190′ $\rightarrow p = 22.7(-.5) = -11.4^{psf}$

Normal to 62′ $\rightarrow p = 22.7(-.25) = -5.69^{psf}$

Roof

distance from windward edge: $0\text{-}h \rightarrow C_p = -.9, -.18$

$h\text{-}2h \rightarrow C_p = -.5, -.18$

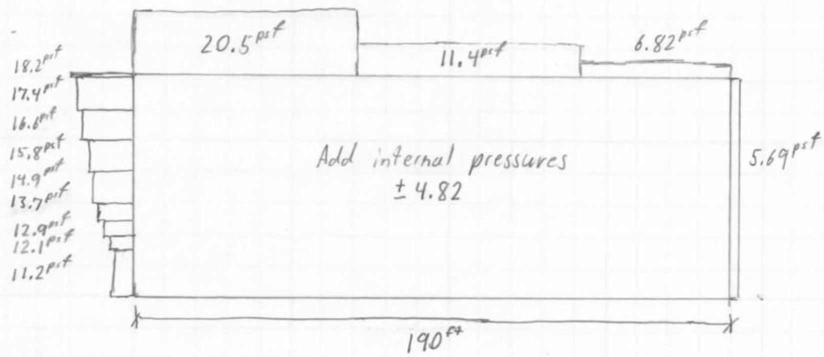
$>2h \rightarrow C_p = -.3, -.18$

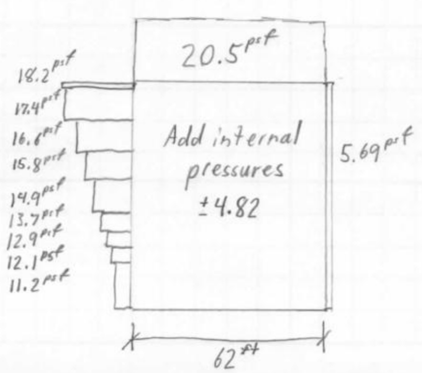
$0\text{-}h \rightarrow p = 22.7(-.9) = -20.5^{psf}$

$= 22.7(-.18) = -4.09^{psf}$

$h\text{-}2h \rightarrow p = 22.7(-.5) = -11.4^{psf}$

$>2h \rightarrow p = 22.7(-.3) = -6.82^{psf}$





Page 2 of 2

Seismic Calcs | Tech 1

Mapped max consideration spectral response acceleration at:

Figure 22-1 → $S_s = 13.5\%$

Figure 22-2 → $S_1 = 5.5\%$

Site Class D

Table 11.4-1 → $F_a = 1.6$

Table 11.4-2 → $F_v = 2.12$

eq 11.4-1 → $S_{ms} = F_a S_s = 1.6(.135) = 21.6\%$

eq 11.4-2 → $S_{m1} = F_v S_1 = 2.12(.055) = 11.7\%$

eq 11.4-3 → $S_{DS} = \frac{2}{3} S_{ms} = \frac{2}{3}(.216) = .144$

eq 11.4-4 → $S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(.117) = .076$

Seismic Design Category per Table 11.6-1 & 11.6-2 → A

Table 12.2-1 → R = 4 light-fram (cold-formed steel) wall systems using flat strap bracing

Table 12.8-2 → $C_t = .02$, $x = .75$

eq 12.8-7 → $T_a = C_t h_n^{\,x} = .02(62^{ft})^{.75} = .44^{sec}$

Table 22-12 → $T_L = 8^{sec} > .44^{sec}$

eq 12.8-2 & 12.8-3 → $C_s = \frac{S_{DS}}{\left(R/I_e\right)} \le \frac{S_{D1}}{T\left(R/I\right)}$

$C_s = \frac{.144}{4} = .036$

$C_s \le \frac{.076}{.44(4)} = .043$

$C_s > .01$

Page 1 of

Seismic Calcs | Tech 1

Dead loads

mechanical equipment

Roof: $W_{rf} = 190^{ft} \times 60^{ft} \times 81.5^{psf} + 7500^{lb} = 936{,}600^{lb}$

Floors: $W_{fl-2} = 190^{ft} \times 60^{ft} \times 81.5^{psf} = 929{,}100^{lb}$

wall weight negligible to floors

$W = 936{,}600^{lb} + 929{,}100^{lb} \times 5$
$= 5{,}582{,}100^{lb}$

eq 12.8-1 → $V = C_s W = .036(5{,}582{,}100^{lb}) = 200{,}956 \approx 201{,}000^{lb}$

Distribution of Seismic Base Shear

eq 12.8-11, 12.8-12 → $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$, $F_x = C_{vx} V$

$\sum_{i=1}^{n} w_i h_i^k = 936{,}600^{lb}(62') + 929{,}100^{lb}(52.5' + 43' + 34' + 24.5' + 13.3') = 2.14 \times 10^8$

$F_{rf} = \dfrac{2.01 \times 10^5}{2.14 \times 10^8}(936{,}600)(62) = 54{,}667^{lb}$

$F_6 = \dfrac{201000}{2.14 \times 10^8}(929100)(52.5) = 45{,}920^{lb}$

$F_5 = \dfrac{201000}{2.14 \times 10^8}(929100)(43) = 37{,}610^{lb}$

$F_4 = \dfrac{201000}{2.14 \times 10^8}(929100)(34) = 29{,}740^{lb}$

$F_3 = \dfrac{201000}{2.14 \times 10^8}(929100)(24.5) = 21{,}429^{lb}$

$F_2 = \dfrac{201000}{2.14 \times 10^8}(929100)(13.3) = 11{,}633^{lb}$

Page 2 of